CONNECTICUT FLOOD PLAIN MANAGEMENT SERVICES

FLOODING ANALYSIS OF THE BLIND BROOK DANBURY, CONNECTICUT

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US Army Corps
of Engineers

New England District

FLOODING ANALYSIS OF THE BLIND BROOK
DANBURY, CONNECTICUT

BY
DEPARTMENT OF THE ARMY
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FLOODING ANALYSIS OF THE BLIND BROOK
DANBURY, CONNECTICUT

1. PURPOSE AND SCOPE

a. General. The purpose of this study was to conduct a hydrologic and hydraulic flood analysis of the Blind Brook watershed located in Danbury, Connecticut. The results of this report can be used by the community to gain a better understanding of flooding along the Blind Brook and possible options for flood control. The Corps of Engineers conducted this study at the request of the City of Danbury, under authority of the Flood Plain Management Services (FPMS) Program. Included in this report are sections describing the watershed, flood history, hydrology, model calibration, hydraulic analysis, and flood reduction options. Plate 1 shows the location of the Blind Brook watershed in Danbury.

2. DESCRIPTION OF WATERSHED

a. General. The Blind Brook is a small stream (drainage area = 1.43 square miles) that flows through the City of Danbury, Connecticut. The brook is a tributary to the Still River, which is a major tributary to the Housatonic River. The Blind Brook begins at the Tarrywile Lake Dam outlet and flows in a northerly direction for one mile where localized stormwater enters the channel at West Wooster Street. Along this reach, the channel width ranges between 2 and 8 feet; with buildings located along the channel edge.

The Blind Brook flows through several culverts that extend long distances underground. These culverts have different inlet and outlet conditions and the underground physical dimensions are unknown. The Blind Brook drops approximately 54 feet between Lincoln Avenue and the confluence with the Still River.

b. Upper Watershed. The upper watershed area is sparsely developed with wooded, mountainous terrain and two storage ponds: Tarrywile Lake and Parks Pond (see Plate 1).

(1) Tarrywile Lake Dam. This dam is located approximately 1 mile southeast of the intersection of Route 7 and Interstate 84 in the City of Danbury, Connecticut. The dam is located at latitude 41^0 22.5' N and longitude 73^0 27' W and is on the Parks Pond Brook. The dam is owned by the City of Danbury and impounds water for recreational purposes only. The watershed above the dam has a drainage area of 0.5 square miles (320 acres) is predominantly undeveloped lands with approximately 5 percent as natural storage and residential development along the eastern shore of Tarrywile Lake. The topography is hilly with elevations ranging from 950 feet NGVD (National Geodetic Vertical Datum of 1929) to 488 NGVD at the spillway crest.

The dam is a cyclopean masonry/earth dam approximately 180 feet long, 12 feet high, and with a 30.5-foot long spillway. There is a lower gatehouse that controls a discharge pipe passing through the base of the dam. The dam crest elevation is 490.2 feet NGVD. The outlet structure was originally a rectangular sluice, which is now closed off and inoperable. There is no other means to drain or regulate the impoundment behind this dam.

(2) Parks Pond Dam. Parks Pond dam is located adjacent to the Tarrywile Lake dam at latitude 41°22' and longitude 73°27'. The watershed area above the dam is 0.20 square miles (128 acres) and is almost entirely undeveloped with topography and elevations ranging between 950 feet NGVD at Thomas Mountain to 538.4 feet NGVD at the spillway crest. The spillway is approximately 20 feet long and the dam crest elevation is 539.7 feet NGVD. Parks Pond dam was rebuilt by the City of Danbury in 1988.

c. Lower Watershed. The Blind Brook watershed downstream of Parks Pond dam and Tarrywile Lake dam includes a total area of 0.73 square miles (466 acres). The lower watershed contains drainage areas that are both low-density residential with natural stormwater attenuation and heavily urbanized with minimal or no stormwater attenuation (see the lower watershed location map on Plate 2).

The drainage area immediately downstream of the dams (sub-drainage areas 3 & 4) includes approximately 0.56 square miles (360 acres) and is a low-density residential area. There are two natural storage areas located in this area that assist in regulating the stormwater runoff generated within this area prior to discharging to the Blind Brook. The first of these natural storage areas is a wetland area upstream of Jefferson Avenue and the second is a pond upstream of West Wooster Street.

The Blind Brook watershed between West Wooster Street and the confluence of the Blind Brook with the Still River (sub-drainage areas 5, 6 & 7) contains approximately 0.169 square miles (108 acres). This area is heavily urbanized and steeply sloped on either side of the Blind Brook. The stormwater runoff generated within this area does not have a source of attenuation to regulate flow; therefore, the stormwater flows rapidly overland to the Blind Brook during a rain event.

3. FLOOD HISTORY

a. General. Though floods can occur any season of the year, historically, most high frequency storms have occurred as a result of hurricanes and tropical storms that hit the Connecticut coast. Major flooding occurred along Blind Brook as a result of storm events in the 1930s, 1950s and most recently, September 1999.

b. September 1999. Total rainfall amounts resulting from the September 16, 1999 storm event were recorded between 7 and 11 inches for western Connecticut. A site visit was made on September 22 to estimate the high water marks produced during this storm event. See Plates 6A, 6B, and 6C for the estimated high water marks and approximate flooded area based on observations made during the site visit and information provided by city engineers.

No organized streamflow data was documented for the Blind Brook during this storm event. The hurricane Floyd rainfall data utilized in this study was provided by NOAA and totaled 9.66 inches of rain over a 24 hour period, see table below.

Table 1
September 16, 1999 Storm Event
Hourly Rainfall Recorded at Brookfield, Ct. Middle School

Time (hrs)	Rainfall (inches)	Time (hrs)	Rainfall (inches)
00:00	0.00	13:00	0.33
01:00	0.02	14:00	0.58
02:00	0.00	15:00	0.62
03:00	0.05	16:00	0.43
04:00	0.04	17:00	0.71
05:00	0.05	18:00	1.07
07:00	0.08	19:00	1.50
08:00	0.11	20:00	1.14
09:00	0.27	21:00	1.03
10:00	0.31	22:00	0.41
11:00	0.34	23:00	0.04
12:00	0.52	24:00	0.01

The estimated flood limits along Blind Brook were predicted from the high water marks recorded during the site visit. Flooding occurred at Montgomery Street and flowed over the large parking lot between Montgomery and New Streets resulting in 1-foot of flooding at the City's New Street Fire Department headquarters. The flooding at Montgomery Street appeared to have been caused by an undersized culvert that extends from Montgomery Street to the far side of New Street. The undersized inlet had a few feet of water over it, which then backed up water over West Street and upstream to George Street. The flooding continued to back up along the brook with approximately 2 feet of water at Williams Street and up to 4 feet of water at E. Pearl Street. There was minor flooding over Jefferson Avenue due to the small restrictive downstream channel located between George Street and Lincoln Avenue.

4. BACKGROUND INFORMATION

a. Prior Studies by the Corps of Engineers. The Corps conducted a study of the Blind Brook in 1986 to determine if it met the criteria for Federal involvement in a flood damage reduction project. The investigation determined that the brook has a 10-year (yr) peak discharge in the range of 200 cubic feet per second (cfs) to 370 cfs. The flow requirement for Corps involvement in flood damage reduction improvements is a 10-yr discharge of 800 cfs or greater (as per ER 1165-2-21). Therefore, further investigation was not warranted. The results of this technical investigation further reinforce that conclusion (i.e. the brook still does not meet the minimum flow criteria).

The Corps (July 1980) also conducted a hydrologic study of Tarrywile Lake dam entitled, "Phase I Inspection Report – National Dam Inspection Program". This study determined the 100-yr inflow to be 197 cfs and the spillway capable of discharging 200 cfs. This study did not route the storm through the lake or provide any detailed analysis of downstream flood conditions.

b. Other Prior Studies. Philip W. Genevese & Associates, Inc, completed a Phase II Dam Inspection Report for Tarrywile Lake dam in April 1984. This study routed both 100-yr and 500-yr storms through the lake using various hydrologic methods. The 100-yr inflow rates ranged between 182 cfs and 432 cfs and the 500-yr inflow rates ranged between 323 cfs and 540 cfs. The results of this study determined a ½ probable maximum flood (PMF) to be 650 cfs, which will overtop the dam by 1.8 inches, or reach an elevation of 490.35 feet NGVD. This study used an SCS curve number (CN) of 66.

Roald Haestad, Inc. analyzed the Parks Pond dam spillway and freeboard for hydraulic adequacy in 1986. This study calculated return frequencies of 2, 10, 25, 50 and 100 years using TR-20 and the HEC-1 computer models. This study determined the spillway capacity to equal a 65-yr storm event with no freeboard.

5. STUDY PROCEDURE

a. General. No organized streamflow records were available for the Blind Brook. However, several hydrologic studies previously discussed were used as references and, along with the September 1999 hurricane rainfall data and high water marks, were used to calibrate the Corps of Engineers hydrologic (HEC-HMS) and hydraulic (HEC-RAS) models.

b. HEC-HMS. The HEC-HMS model was used to simulate reservoir storage routings and calculate overland stormwater runoff. Storage routings are based on the continuity equation (inflow = outflow + change in storage). Input for the model consisted of storage characteristics for Parks Pond and Tarrywile Lake, inflow hydrographs, and spillway discharge characteristics. Runoff for inflow into the ponds as well as the downstream contribution drainage areas was based on drainage area size, slope, and land use characteristics.

The Blind Brook drainage area was sub-divided into 7 sub-basins. See Plates 1 and 2 for the sub-drainage locations and Plate 3 for the watershed schematic. Soil Conservation Service (SCS) dimensionless unit hydrographs based on the time of concentration were used to represent runoff regimes for each sub-basin. Loss rates were based on SCS curve numbers (CN) that represent land use in each sub-basin. The lag time, a weighted time of concentration dependent upon physical properties of the watershed, was calculated for each subbasin using the SCS lag time equation.

A tabulation of SCS curve numbers, loss rates, lag time, area, and the base flow rates are given in Table 2.

Table 2
Sub-Watershed Hydrologic Characteristics

Sub-Area	Watershed Area Sq.Mi (Ac.)	Curve No. (CN)	Lag Time (Min.)	Baseflow (cfs)	% Impervious
1- Tarrywile Lake Dam	0.5 (320)	55	41	5	3
2- Parks Pond Dam	0.2 (128)	50	27	5	0
3	0.35 (224)	62	20	10	30
4	0.21 (134)	60	20	10	25
5	0.095 (61)	90	12	10	75
6	0.034 (22)	90	10	10	75
7	0.040 (25)	75	16	10	35

(1) Upper Watershed Analysis. Area-capacity relationships were determined from calculated Parks Pond and Tarrywile Lake surface areas at normal pool, and a contour line approximately five feet above the water surface elevation, from USGS Quad Sheets. See Tables 3 and 4 for the adopted storage-discharge relationships used in this study. Spillway discharge ratings were developed using the weir equation with adopted weir flow coefficients of 2.65 for the spillway. The adopted spillway rating curves are presented in Plates 4 and 5.

The September 1999 flood hydrographs were computed based on the above hydrologic characteristics and the rainfall shown in Table 1. Hydrographs were then routed through the Tarrywile Lake and Parks Pond dams to calibrate the HEC-HMS model. As noted previously, the Phase I inspection report conducted for Tarrywile Lake dam used a CN of 66. Assuming the pool elevation is at the spillway crest and the watershed reflects a CN of 66, HEC-HMS calculated the September 1999 flood frequency discharge of 286 cfs, which would have overtopped the Tarrywile Lake dam by 4 inches. Due to the fact the Tarrywile Lake dam has never been reported as being overtopped, curve numbers of 51 and 55 have been determined representative of the Parks Pond and Tarrywile Lake watersheds at the time of this study.

Table 3
Parks Pond
Stage-Storage Properties
(Dam crest elev. 539.7 feet)

Elevation (Ft. NGVD)	Storage (Ac-Ft)	Outflow (cfs)
538.4	0.0	0
539.0	2.0	26
540.0	8.0	230
545.0	44.0	1302

Table 4
Tarrywile Lake
Stage-Storage Properties
(Dam crest elev. 490.2 feet)

Elevation (Ft. NGVD)	Storage (Ac-Ft)	Outflow (cfs)
488.0	0.0	0
488.5	10.0	30
489.0	23.0	80
489.5	40.0	150
490.0	50.0	230

The calculated Parks Pond and Tarrywile Lake peak outflow rates during the September 1999 flood event were 96 cfs and 205 cfs, respectively. This corresponds to 1.40 feet above the Parks Pond Dam spillway crest and 1.85 feet above the Tarrywile Pond Dam spillway crest, which is in general agreement with reports from locals concerning the September 1999 flood.

(2) Lower Watershed Analysis.

Area-capacity relationships were computed for the natural attenuation areas located upstream of both Jefferson Avenue and West Wooster Street. Topographic maps provided by the City of Danbury were used to estimate the storage area at 2-foot contour intervals.

The elevation-outflow relationships for these two storage areas were calculated using the HEC-RAS model since water surface elevations at West Wooster Street and Jefferson Avenue are impacted by downstream backwater from the Blind Brook. Random flow rates were input into the HEC-RAS model, which was developed from survey data and expected culvert and channel characteristics, to determine the corresponding upstream water surface elevations. These computed elevation-outflow relationships were then input into the HEC-HMS model which defined the flow characteristics of the September 1999 storm needed to calibrate the HEC-RAS model. Refer to Section 5.c. for discussion of the HEC-RAS model. See Tables 5 and 6 for the adopted elevation-storage-outflow relationships used for these areas.

After the HEC-HMS model was calibrated, 2-, 5-, 10-, 25-, 50-, and 100-yr storm events were computed and routed through the Blind Brook watershed. The rainfall data used to calculate these storm events was provided by the U.S. Weather Bureau Technical Paper No. 40 (TP-40) and shown in Table 7. The 12-hour storm duration was used because it had been used in previous studies and was believed to be a reasonably severe condition. The results of these simulated storm events are presented in Table 8. See Appendix A for the HEC-HMS summary of calculated flow rates for various storm events.

Table 5
Upstream of West Wooster Street
Stage-Storage Properties

Elevation (Ft. NGVD)	Storage (Ac-Ft)	Outflow (cfs)
423.5	0.36	7
424.0	0.73	14
424.5	2.6	22
425.0	4.5	31
425.5	7.9	40
426.0	11.4	140
426.5	17.0	240
427.0	22.7	500
427.5	28.4	1000

Table 6
Upstream of Jefferson Avenue
Stage-Storage Properties

Elevation (Ft. NGVD)	Storage (Ac-Ft)	Outflow (cfs)
452.0	0.0	0
452.5	0.91	40
453.0	3.3	65
453.5	4.9	110
454.0	6.5	185
455.0	9.8	500
456.0	13.0	930

Table 7
Technical Paper No. 40 Data
(Rainfall Depth in inches)

Time (hrs)	2yr/ 12hr	5yr/ 12hr	10yr/ 12hr	25yr/ 12hr	50yr/ 12hr	100yr/ 12hr
1	1.3	1.7	2.0	2.3	2.5	2.8
2	1.6	2.1	2.5	2.8	3.3	3.5
3	1.8	2.3	2.8	3.3	3.5	4.0
6	2.3	3.0	3.4	4.0	4.8	5.0
12	3.3	3.6	4.2	4.8	5.8	6.0

Table 8
Peak Discharge (cfs) Results

Watershed	Area Sq.Mi.	2yr/12hr (cfs)	5yr/12hr (cfs)	10yr/12hr (cfs)	25yr/ 12hr (cfs)	50yr/ 12hr (cfs)	100yr/ 12hr (cfs)
1-Parks Pond Outflow	0.2	7	9	16	25	48	58
2-Tarrywile Lake Outflow	0.5	10	17	28	43	65	73
Junc. 1	0.7	17	26	41	63	97	111
D. A. 3	0.35	105	165	223	280	344	381
Upper Jefferson Ave. Outflow	1.05	84	148	221	291	368	406
D. A. 4	0.21	56	89	121	153	191	212
Upper West Wooster Outflow	0.21	27	35	48	86	128	144
Junc. 2-West Wooster St.	1.26	110	181	258	354	482	542
D. A. 5	0.095	77	105	127	146	162	177
D. A. 6	0.034	34	44	51	58	64	69
Junc. 3- West St.	1.389	194	291	389	505	651	720
D.A. 7	0.040	27	37	45	53	61	66
Junc. 4- Downstream Limit	1.43	221	327	432	558	709	786

*Note: See Plate 3 for Watershed Schematic

c. <u>HEC-RAS</u>. The HEC-RAS model was used to simulate downstream flood levels during various storm events. This model calculates the water surface elevations (WSE) along a river channel using the standard step method. The results and effects of backwater along the channel are a function of several parameters used in the HEC-RAS program including flow rate, channel cross-section data, initial downstream WSE, and channel and overbank roughness factors.

As mentioned previously, the Blind Brook has several culverts that stretch long distances underground with different inlet and outlet dimensions. The points of transition along the underground culverts is unknown; however, a survey of the culverts was conducted to provide accurate inlet and outlet dimensions and invert information. The HEC-RAS culvert function assumes the entire culvert length is uniform and, therefore, does not allow for varying the inlet and outlet culvert dimensions. For the purposes of this study, the culvert inlet or outlet of lesser cross-sectional area was input into the model.

The backwater analysis started approximately 500 feet upstream from the confluence of the Blind Brook and the Still River, where a culvert exits adjacent to the shelter at Elm Street. Approximately 55 cross-sections and 11 culverts were input into the model along the 0.80-mile channel. Cross-sections were input at the culvert inlets and outlets as well as 20-feet upstream and downstream of each culvert inlet and outlet. The Blind Brook winds through an urbanized area with buildings located along the channel edge, which makes it difficult to model. Cross-sections were input where buildings located adjacent to the channel were expected to restrict overbank flow. See Plates 6A, 6B and 6C for location of cross sections.

Once the mixed-flow model was developed, the computed 1999 storm event flow rate was used to calibrate the hydraulics based on the estimated high water marks. Based on observed field conditions, the water surface elevation at the downstream end of the study was 388 feet NGVD and the upstream starting water surface elevation was 442 feet NGVD. Estimated water surface elevations throughout the study reach, based on observed water depths during the 1999 storm event, are shown on Plate 7.

The model was run iteratively, adjusting the cross-sectional parameters until the water surface elevations along the channel represented the high water marks observed shortly after the September 1999 storm event. The Manning's 'n' value of the overbank areas ranged between 0.015 to 0.10 and ranged between 0.015 to 0.03 in the channel; however, where debris was reported as blocking the channel, 'n' values of 0.2 and 0.3 were used. The model was calibrated within 0.5 feet of the estimated high water marks. See Plate 7 for the calibrated water surface profile.

The 100-yr frequency peak discharge rate determined by the HEC-HMS model was very similar to the peak discharge determined for the September 1999 storm. Therefore, the HEC-RAS backwater analysis for the 100-year flood event is similar to the flood patterns of the September 1999 flood event.

6. STUDY RESULTS

a. General. The steep topography and urban development associated with the Blind Brook watershed generates high flow rates for high frequency storm events. The flow rates analyzed ranged from a maximum 2-yr flow rate of 220 cfs to a maximum 100-yr flow rate of 786 cfs, which inundates the entire brook. The flow rates calculated by the model defined the channel and culvert characteristics necessary to identify options for flood control improvements.

Plate 8 presents the 10-yr flood hydrograph development within the Blind Brook watershed. As can be seen, the localized sub-drainage areas (5, 6, & 7) peak first, immediately followed by the outflow from the upper Jefferson Avenue storage area and the upper West Wooster Street storage area. Plate 9 presents the total 10-yr hydrographs at various locations along the brook.

As computed by the HEC-HMS model, Tarrywile Lake and Parks Pond dams had maximum releases of 28 cfs and 16 cfs, respectively, during a 10-yr/12-hr storm event and 73 cfs and 58 cfs, respectively, during a 100-yr/12-hr storm event. As presented in both Plate 8 and Plate 9, the time-to-peak outflow from the dams occurs nearly 2 hours after the lower watershed peaks. Therefore, the flow rate overtopping the Parks Pond and the Tarrywile Lake spillways is not a major contribution to downstream flooding in comparison to the flow rates generated in the lower watershed.

The culverts along the Blind Brook were hydraulically modeled at multiple flow rates to determine the water surface elevation for a range of events. Plate 10 presents the water surface profiles for 2-, 10-, and 100-yr storm events. As can be seen, the capacity of the brook is less than a 2-year storm event. The backwater reaches the West Wooster Street roadway crest elevation of 425.8 ft at a flow rate of 140 cfs for the entire watershed. In addition to the under capacity channel and restricting culverts, the low sloped channel between George Street and West Wooster Street exacerbates the backwater along the brook.

b. Flood Improvement Options.

(1) Upper Watershed. Due to the low outflow rates and delay in time-to-peak, which desyncronizes peak outflow with downstream peak runoff, increasing the storage at Parks Pond or Tarrywile Lake would not result in a significant reduction in downstream flooding within the Blind Brook watershed.

(2). Lower Watershed. The flood reduction options that could reduce downstream flooding may be a combination of controlled storage in the natural attenuation areas located upstream of Jefferson Avenue and West Wooster Street and culvert and channel improvements along the Blind Brook between West Street and West Wooster Street.

Currently, backwater from the Blind Brook restricts outflow from the storage areas upstream of Jefferson Avenue and West Wooster Street indicating a significant storage potential within these upstream watersheds. Several storm events were routed through the storage areas with the HEC-HMS model to determine the storage potential of the ponds with a maximum discharge of 50 cfs for each storage area. See Table 9 for the expected water surface elevation for each storm event. This option requires further investigation to determine the type of controlled outlet needed and any improvements required within the existing storage areas, and to insure such measures would not cause a flood problem adjacent to the storage areas. If these options proved feasible they may reduce downstream peak discharges by 380 cfs for the 100-yr event, for example, and result in a stage reduction ranging from 2 to 5 feet along the brook.

Table 9
Estimated Water Surface Elevation
With Increased Storage

Storm Event	WSE* upstream Jefferson Ave.	WSE* upstream West Wooster St.
2-yr	453.4	424.8
5-yr	455.6	425.2
10-yr	457.8	425.6
25-yr	459.3	426.0
50-yr	461.2	426.3
100-yr	461.6	426.5

* WSE – water surface elevation

It also appears that significant flood reduction measures will need to include increasing the channel capacity and channel slope along the Blind Brook. The channel must convey localized flow entering the channel from drainage areas 5, 6, & 7 and any flow released from the controlled upstream storage areas. A 100-yr storm event, with a total localized flow rate of 312 cfs plus 100 cfs from upstream storage areas, will require a minimum channel cross-sectional area of 38 ft^2 and a minimum channel slope of 1.0 % to effectively discharge stormwater entering the Blind Brook. The following conditions may be involved with the culvert and channel improvement option:

a. Channels and culverts that stretch long distances underground may have to be converted to open channels in order to increase the capacity and channel slope. These underground channels have various structures and private property over them, which may require extensive demolition work and real estate acquisitions.

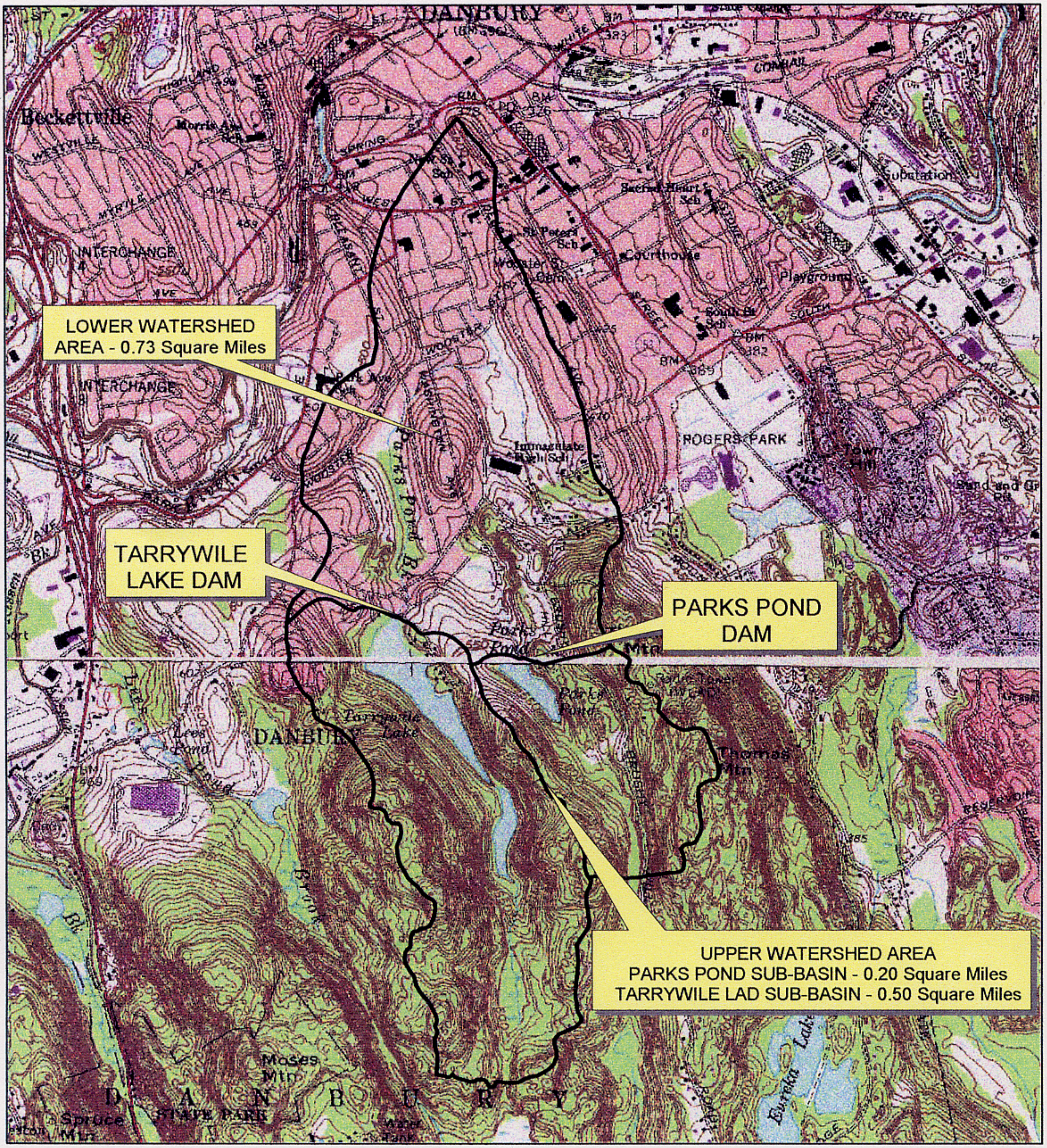
b. This option will require improvements to bridges and culverts located at George Street, William Street, E. Pearl Street, and West Wooster Street; however, roadway work should be minimal assuming the road crest elevation remains unchanged.

c. Increasing the conveyance of the Blind Brook between West Street and West Wooster Street may result in downstream flooding, which would need to be quantified by further study and any required mitigation measures determined.

(3). Estimated Cost of Improvements. An approximate cost estimate of the above-described improvements was developed for the city for planning purposes. This estimate does not include all the efforts that may be required to complete the work (e.g., a feasibility study, real estate acquisitions). The estimate assumed that no special dewatering controls would be required, that all construction access can be obtained, and that there are no hazardous materials at the site. The estimate reflects construction activities along Blind Brook between West Street and the naturally occurring storage areas upstream of Jefferson Street and West Wooster Street.

Table 10
Construction Cost Estimate
For Flood Control Improvements Along Blind Brook

Storage Area Dikes w/control structures: (500 linear feet x 8 feet high each)	
Jefferson Street	$110,000
West Wooster Street	$110,000
Channel Reconstruction: (Assumed 6 feet deep x 7 feet wide precast open channel sections)	
Station 16+40 to 19+08 (Includes demolition of 2 buildings and new box culvert at West Street)	$164,700
Station 19+08 to 24+00 (Includes demolition of 1 building)	$190,000
Station 24+00 to 28+90 (Includes demolition of 2 buildings and new box culvert at George Street)	$190,000
Station 28+90 to 31+55 (Includes new box culvert at William Street)	$ 98,800
Station 31+55 to 36+83 (Includes new box culverts at East Pearl Street and West Wooster Street)	$203,500
Miscellaneous Expenses:	
Chain Link Fence (4100 linear feet)	$ 61,500
Survey Support	$ 50,000
Cost Escalation (3%)	$ 35,400
Contingency (20%)	$235,700
TOTAL COST	**$1,449,600**



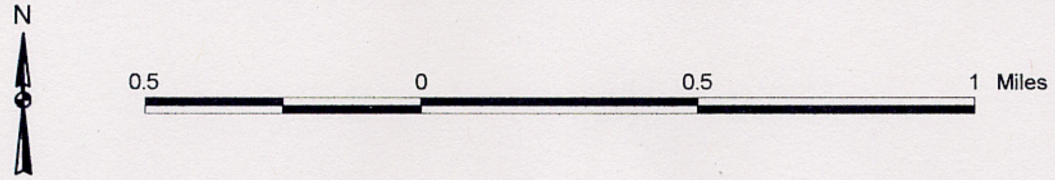
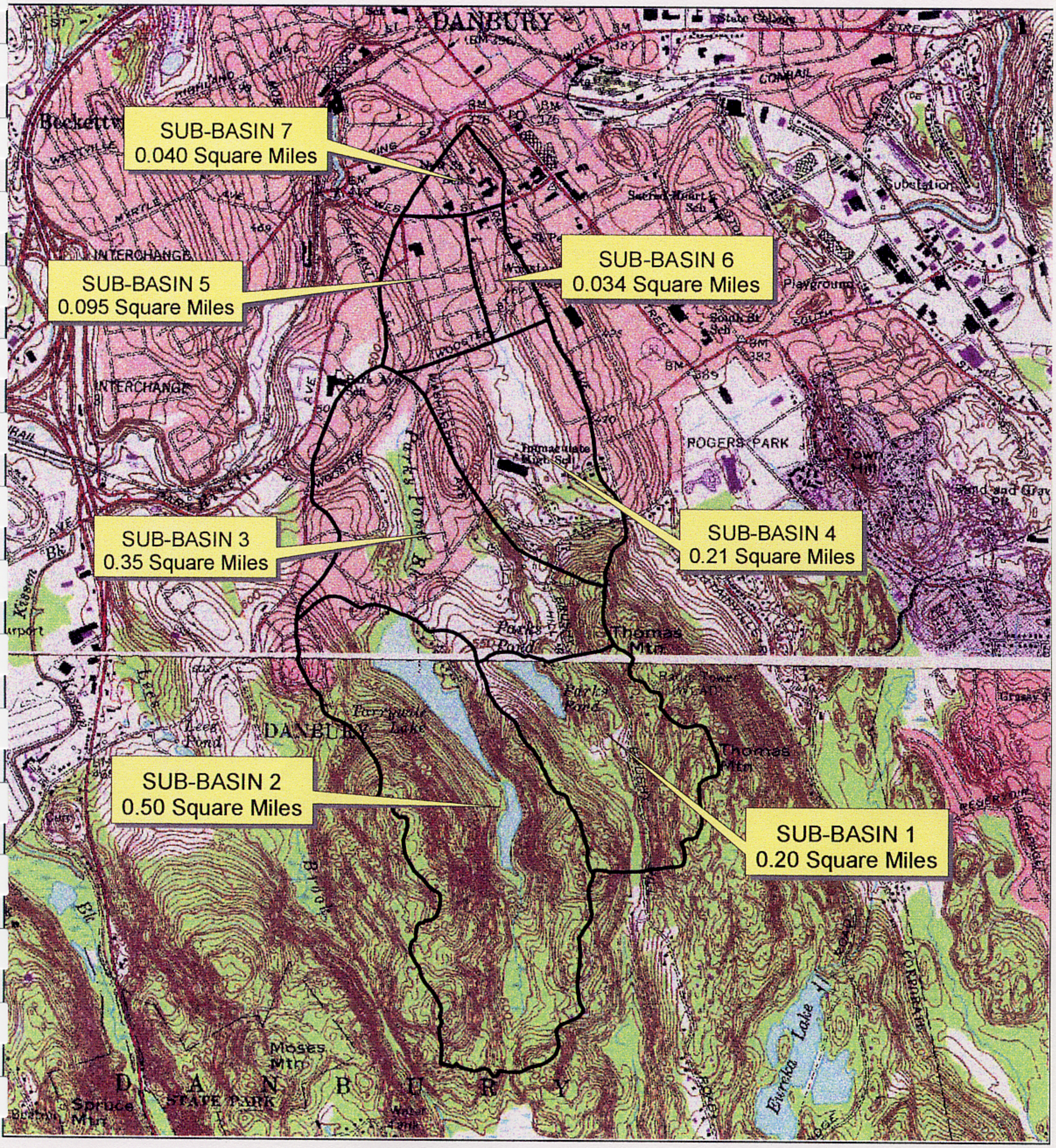


PLATE 1
LOCATION MAP &
BLIND BROOK
WATERSHED MAP
DANBURY, CT



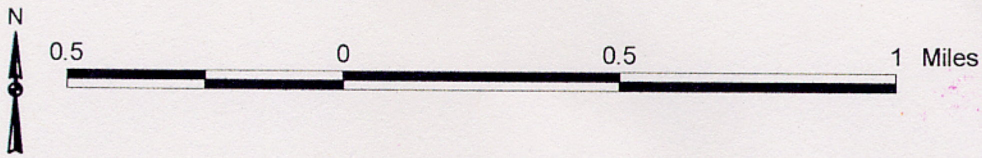


PLATE 2
HEC-HMS SUB-BASINS
BLIND BROOK
DANBURY, CT

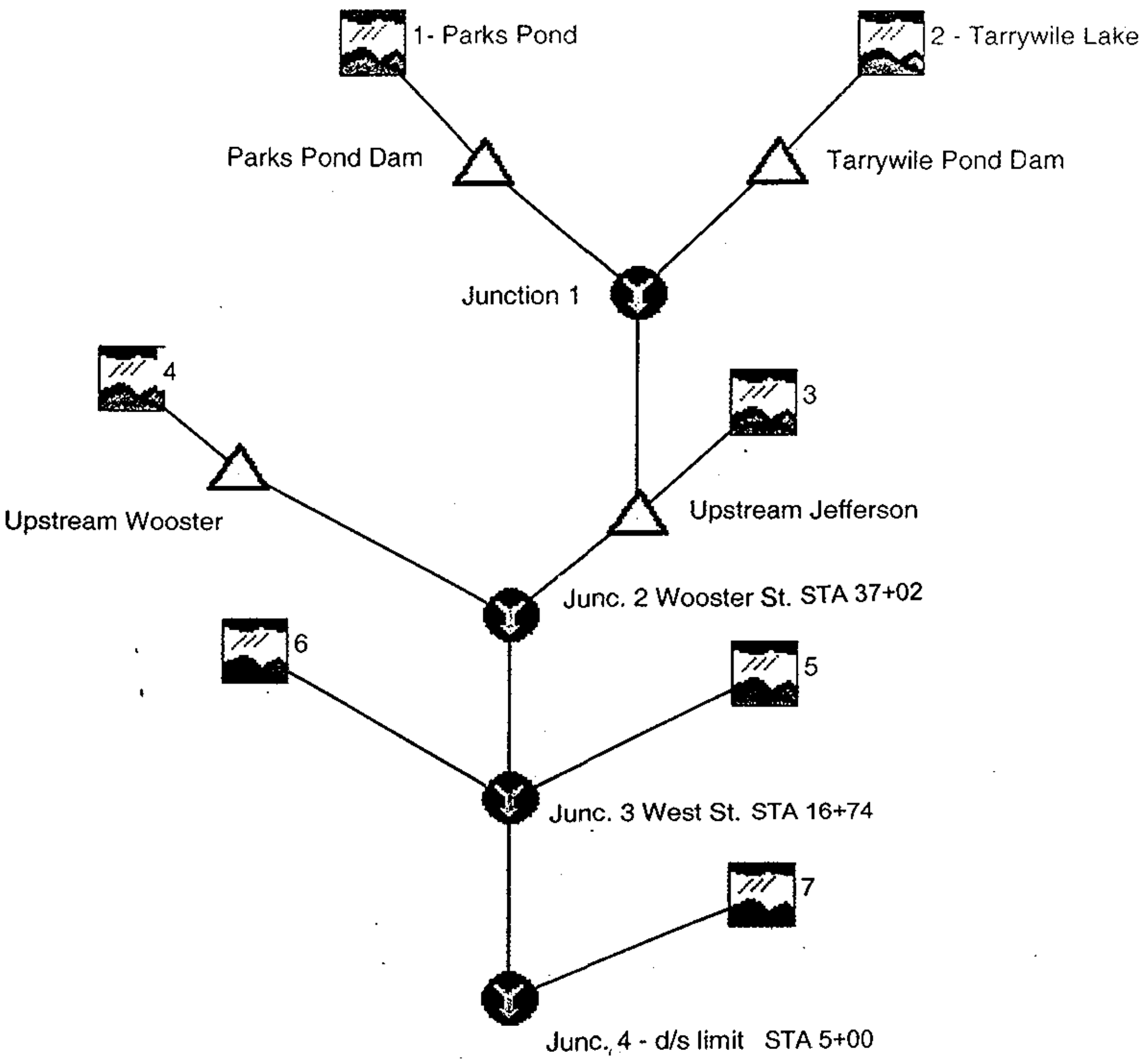


PLATE 3
HEC-HMS WATERSHED SCHEMATIC
BLIND BROOK
DANBURY, CT

PLATE 4
Rating Curve - Tarrywile Lake Dam
Blind Brook - Danbury, Ct.

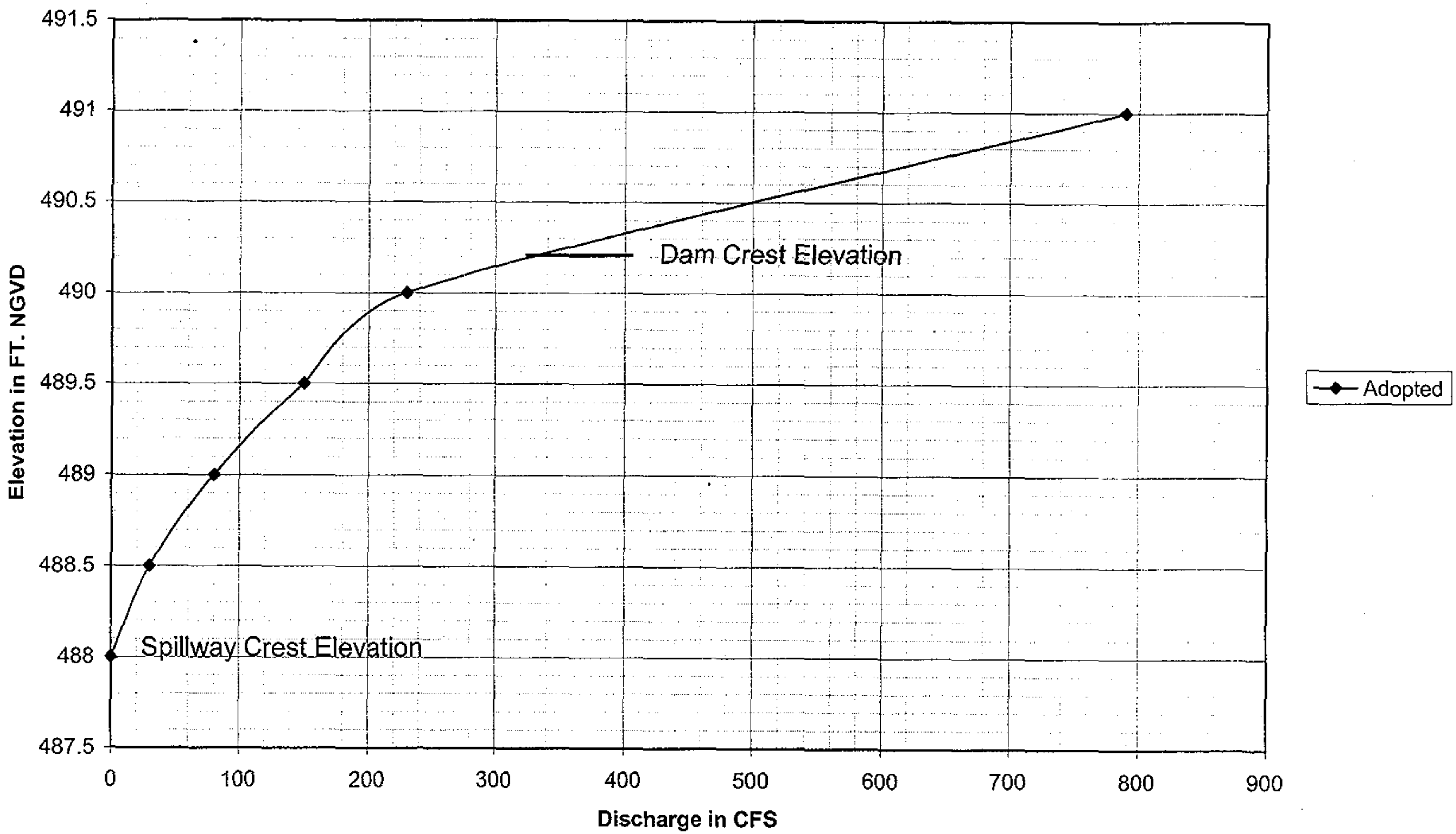


PLATE 4
TARRYWILE LAKE DAM
RATING CURVE

PLATE 5
Rating Curve - Parks Pond Dam
Blind Brook - Danbury, Ct.

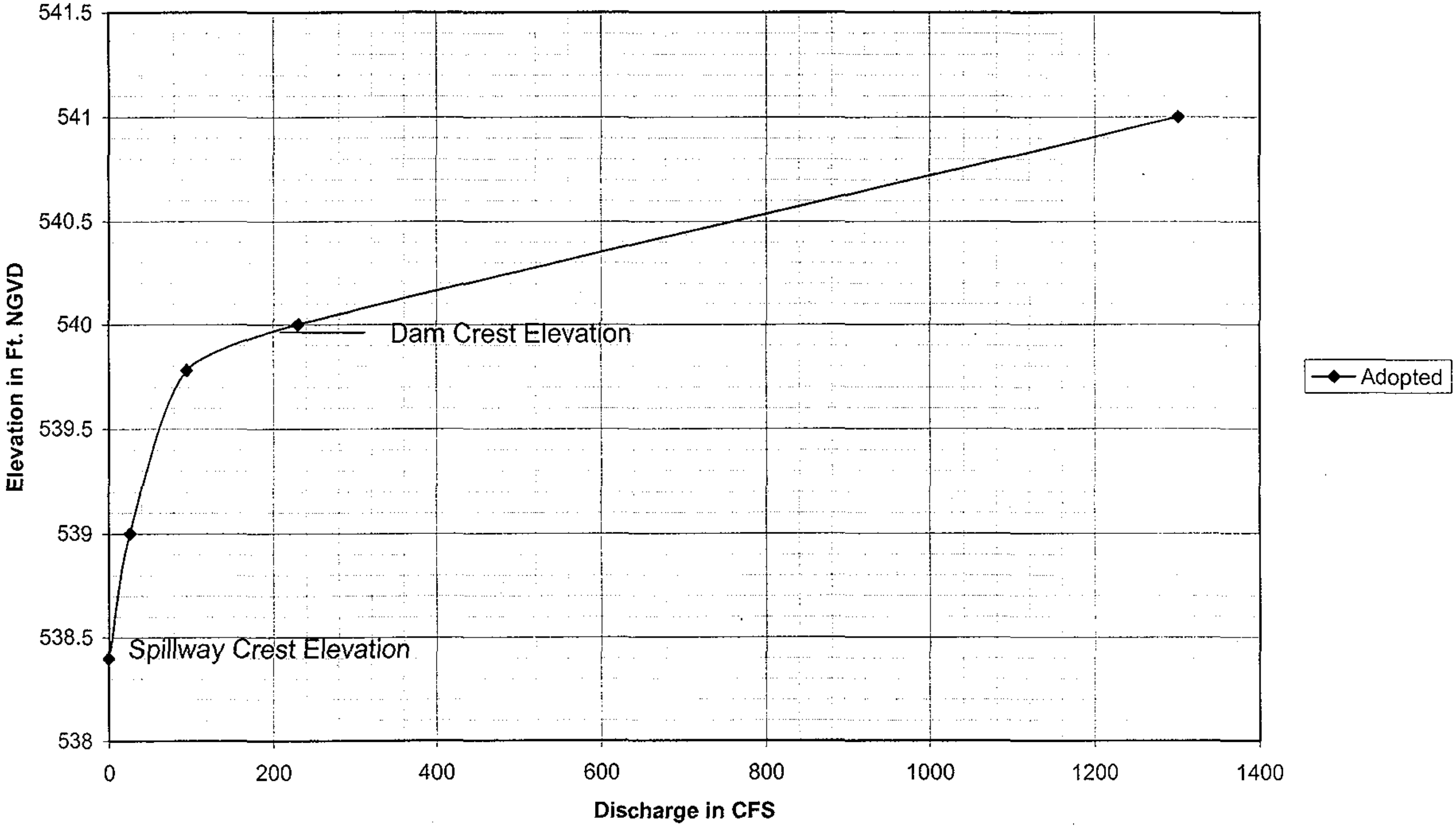
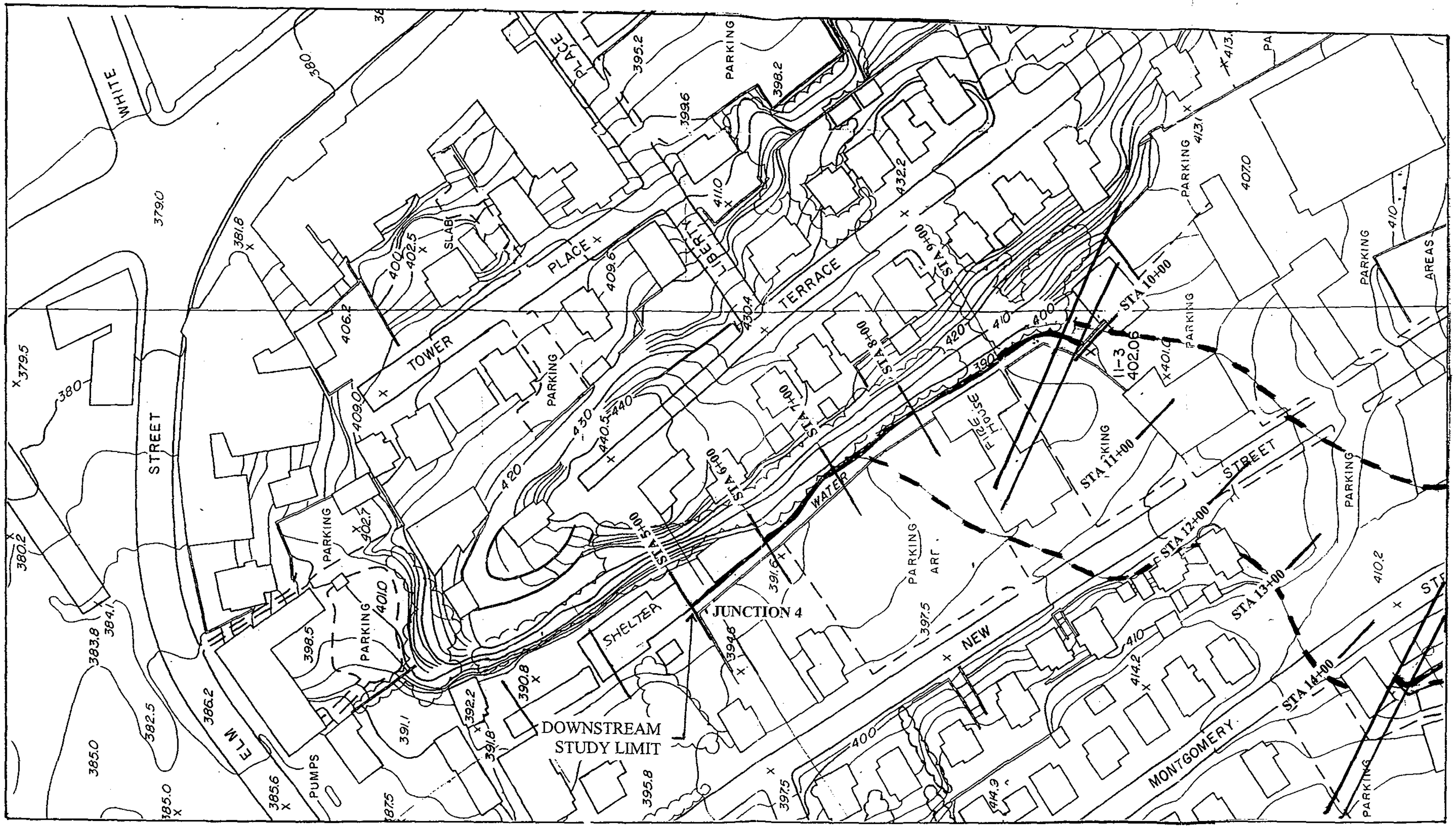


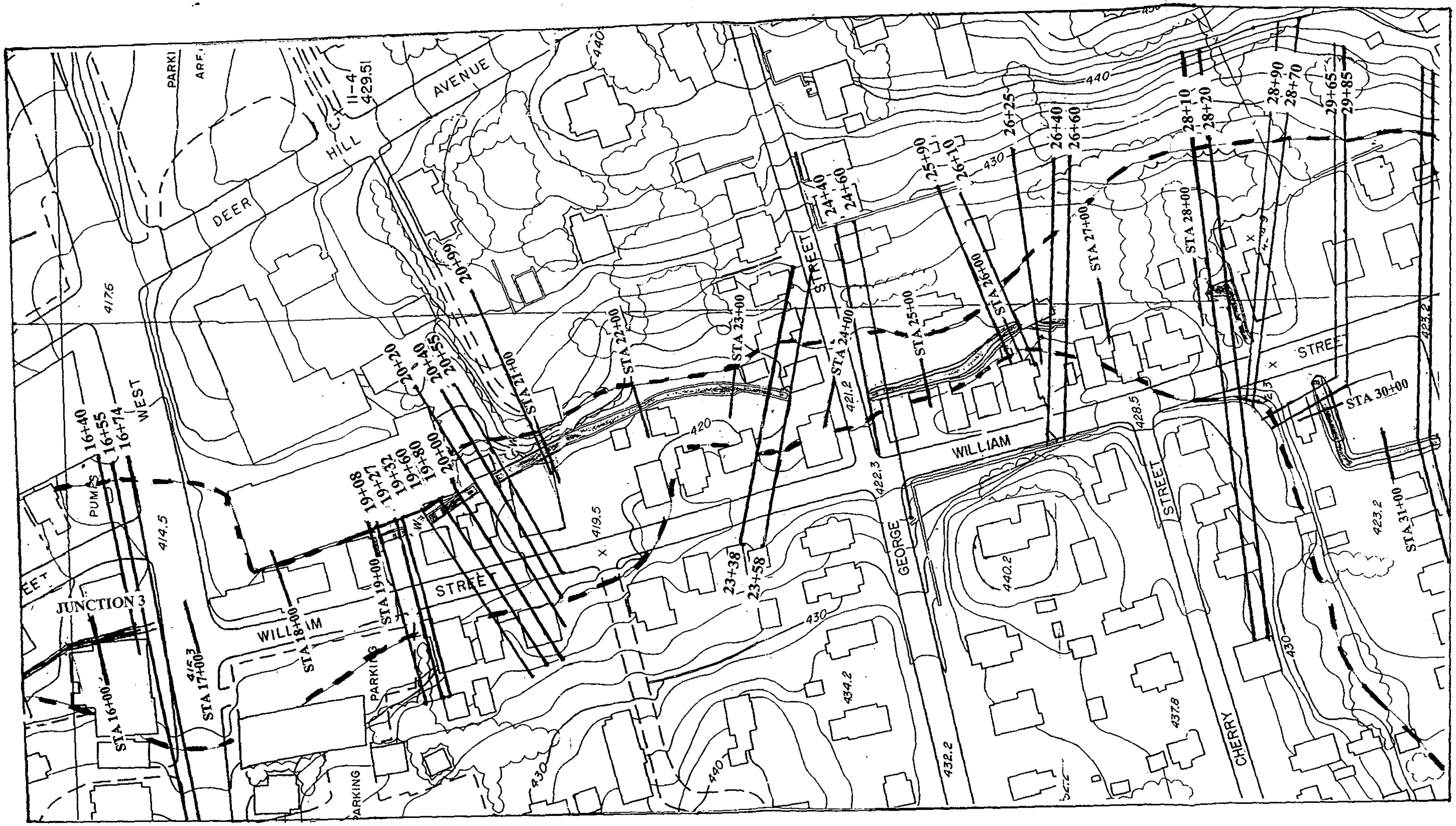
PLATE 5
PARKS POND DAM
RATING CURVE



Legend

Estimated High Water Marks

PLATE 6A
ESTIMATED HIGH WATER MARKS FOR
SEPTEMBER 1999 STORM EVENT AND
HEC-RAS CROSS-SECTION LOCATIONS
BLIND BROOK - DANBURY, CT



Legend

Estimated High Water Marks — — — —

PLATE 6B

ESTIMATED HIGH WATER MARKS FOR SEPTEMBER 1999 STORM EVENT AND HEC-RAS CROSS-SECTION LOCATIONS BLIND BROOK - DANBURY, CT

Legend

Estimated High Water Marks — — -- —

PLATE 6C
ESTIMATED HIGH WATER MARKS FOR
SEPTEMBER 1999 STORM EVENT AND
HEC-RAS CROSS-SECTION LOCATIONS
BLIND BROOK - DANBURY, CT

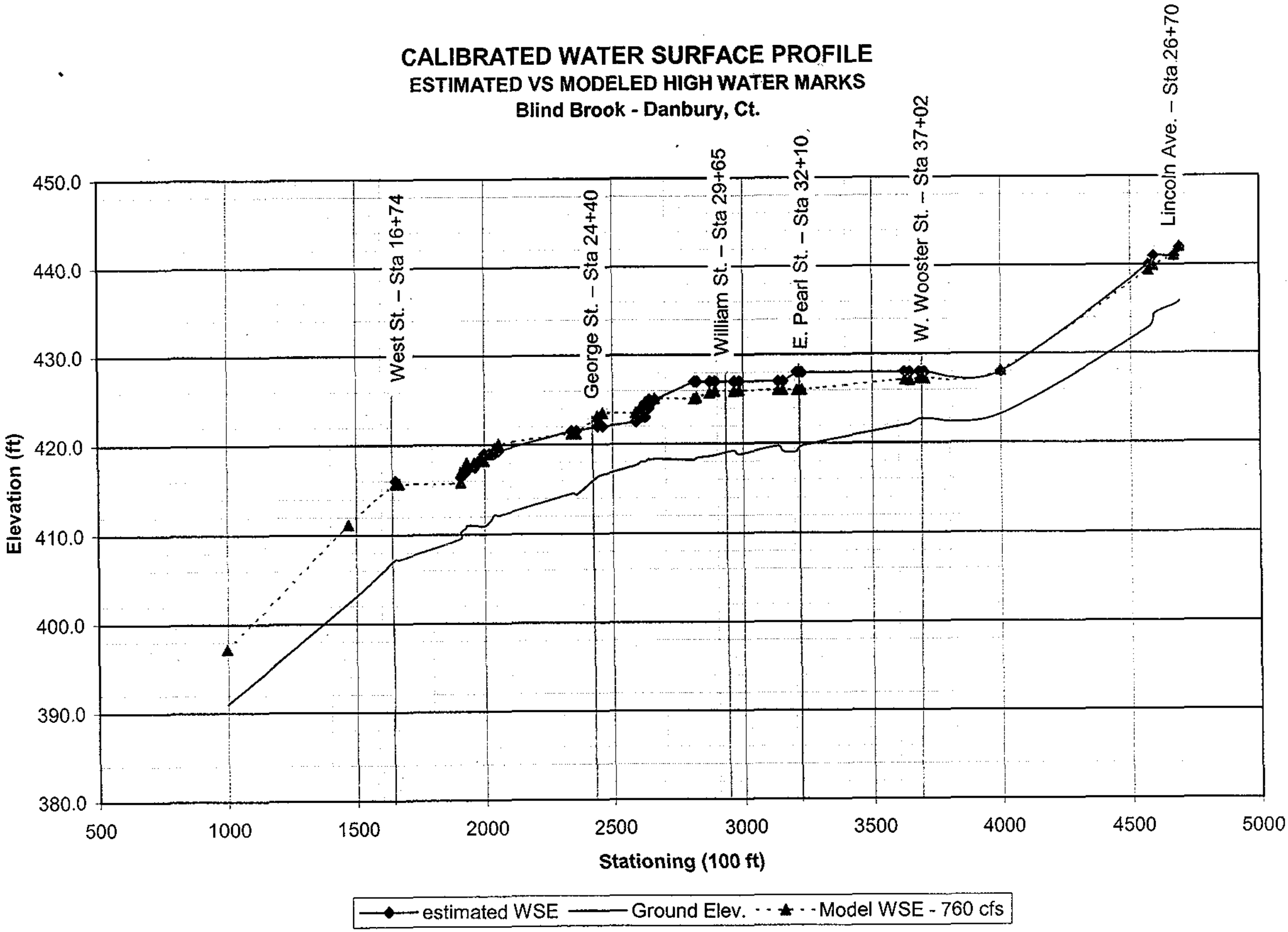


PLATE 7
ESTIMATED HIGH WATER MARKS AND
MODELED WATER SURFACE PROFILE
FOR SEPTEMBER 1999 STORM EVENT

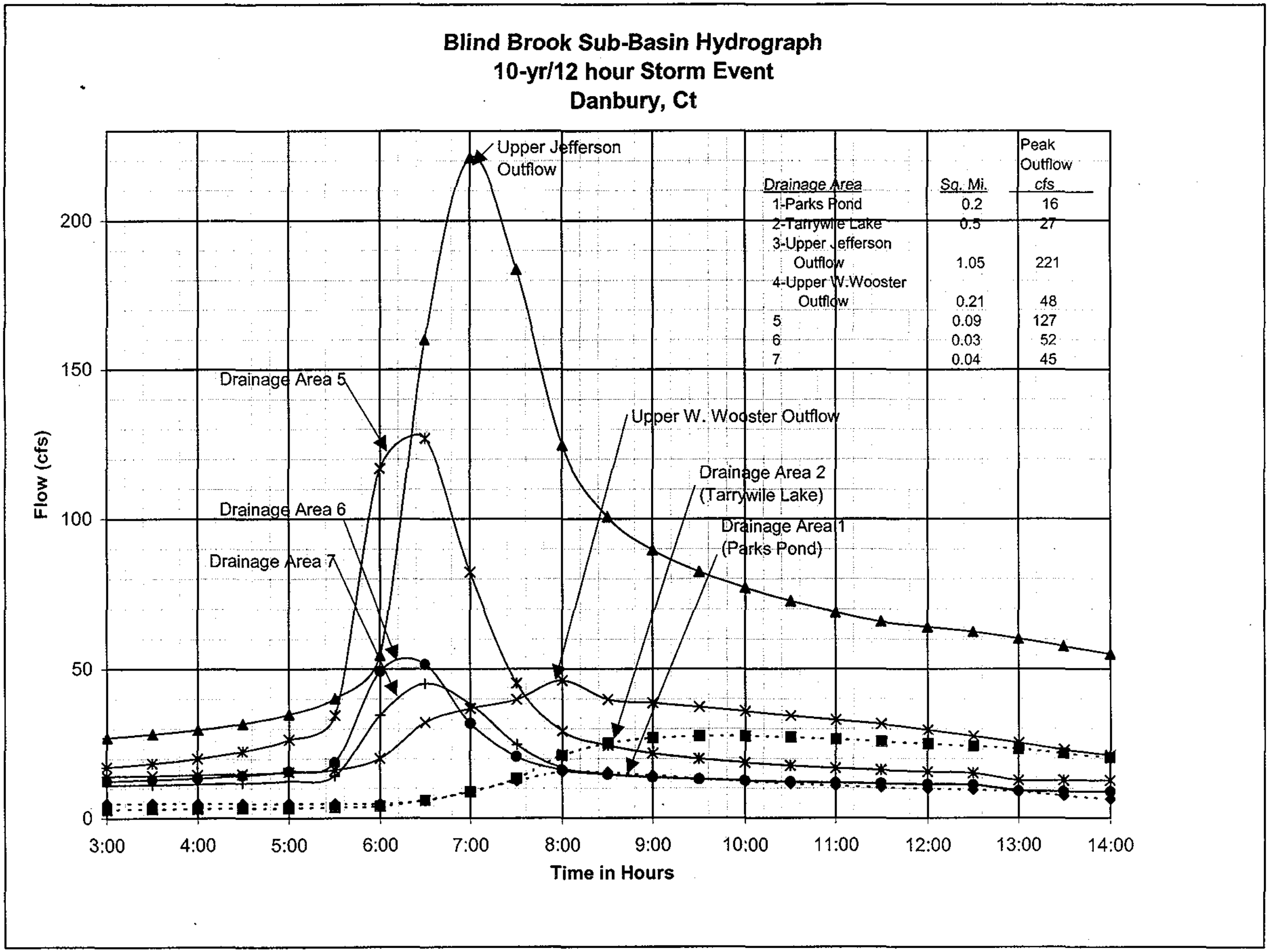
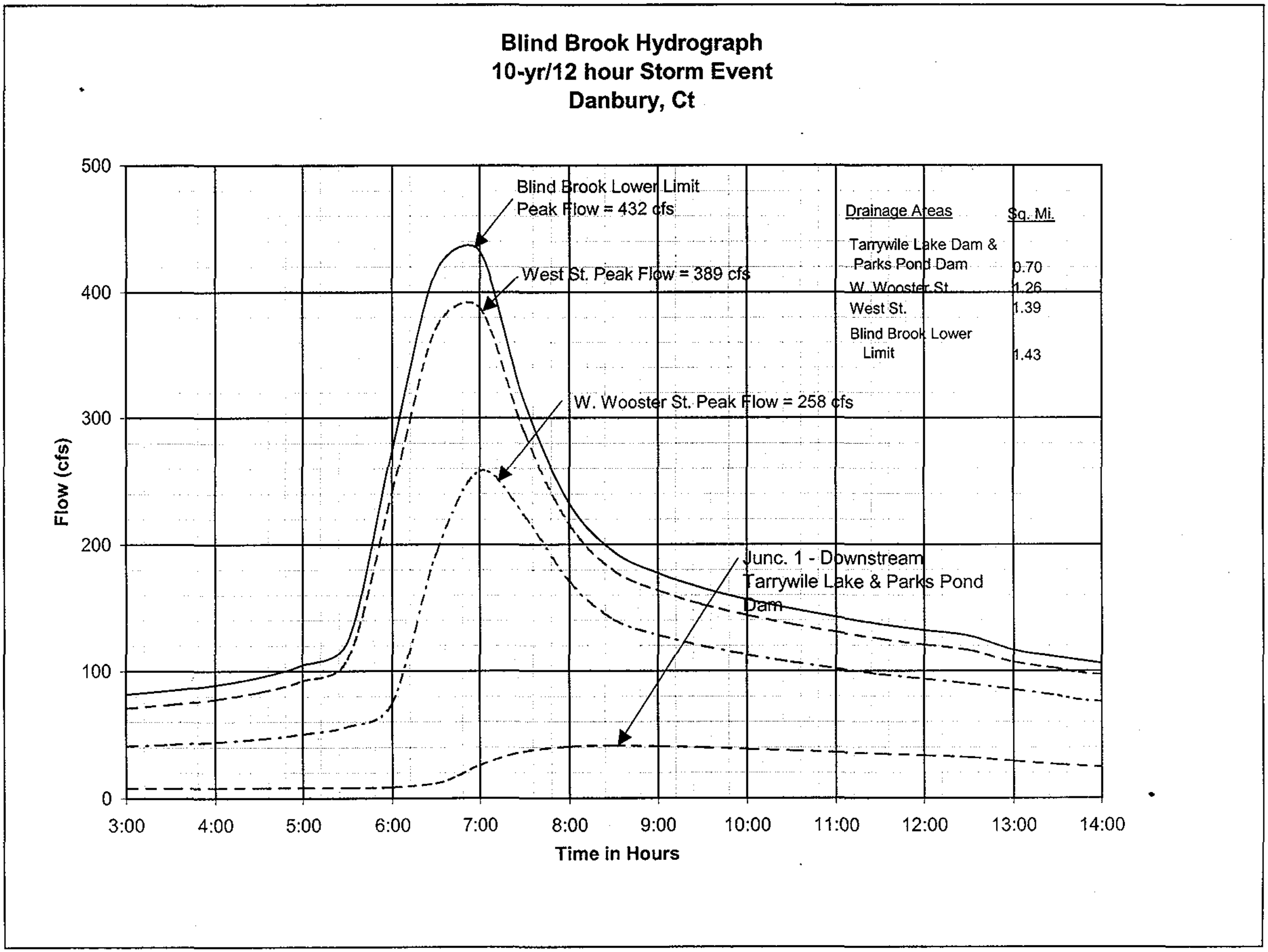

Blind Brook Sub-Basin Hydrograph
10-yr/12 hour Storm Event
Danbury, Ct
Flow (cfs)
Time in Hours
0
50
100
150
200
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00
Upper Jefferson Outflow
Drainage Area 5
Drainage Area 6
Drainage Area 7
Upper W. Wooster Outflow
Drainage Area 2 (Tarrywile Lake)
Drainage Area 1 (Parks Pond)
Drainage Area | Sq. Mi. | Peak Outflow cfs
1-Parks Pond | 0.2 | 16
2-Tarrywile Lake | 0.5 | 27
3-Upper Jefferson Outflow | 1.05 | 221
4-Upper W.Wooster Outflow | 0.21 | 48
5 | 0.09 | 127
6 | 0.03 | 52
7 | 0.04 | 45

PLATE 8



Blind Brook Hydrograph
10-yr/12 hour Storm Event
Danbury, Ct
Blind Brook Lower Limit
Peak Flow = 432 cfs
West St. Peak Flow = 389 cfs
W. Wooster St. Peak Flow = 258 cfs
Junc. 1 - Downstream Tarrywile Lake & Parks Pond Dam
Drainage Areas
Sq. Mi.
Tarrywile Lake Dam & Parks Pond Dam 0.70
W. Wooster St. 1.26
West St. 1.39
Blind Brook Lower Limit 1.43
Flow (cfs)
Time in Hours
500
400
300
200
100
0
3:00
4:00
5:00
6:00
7:00
8:00
9:00
10:00
11:00
12:00
13:00
14:00

PLATE 9

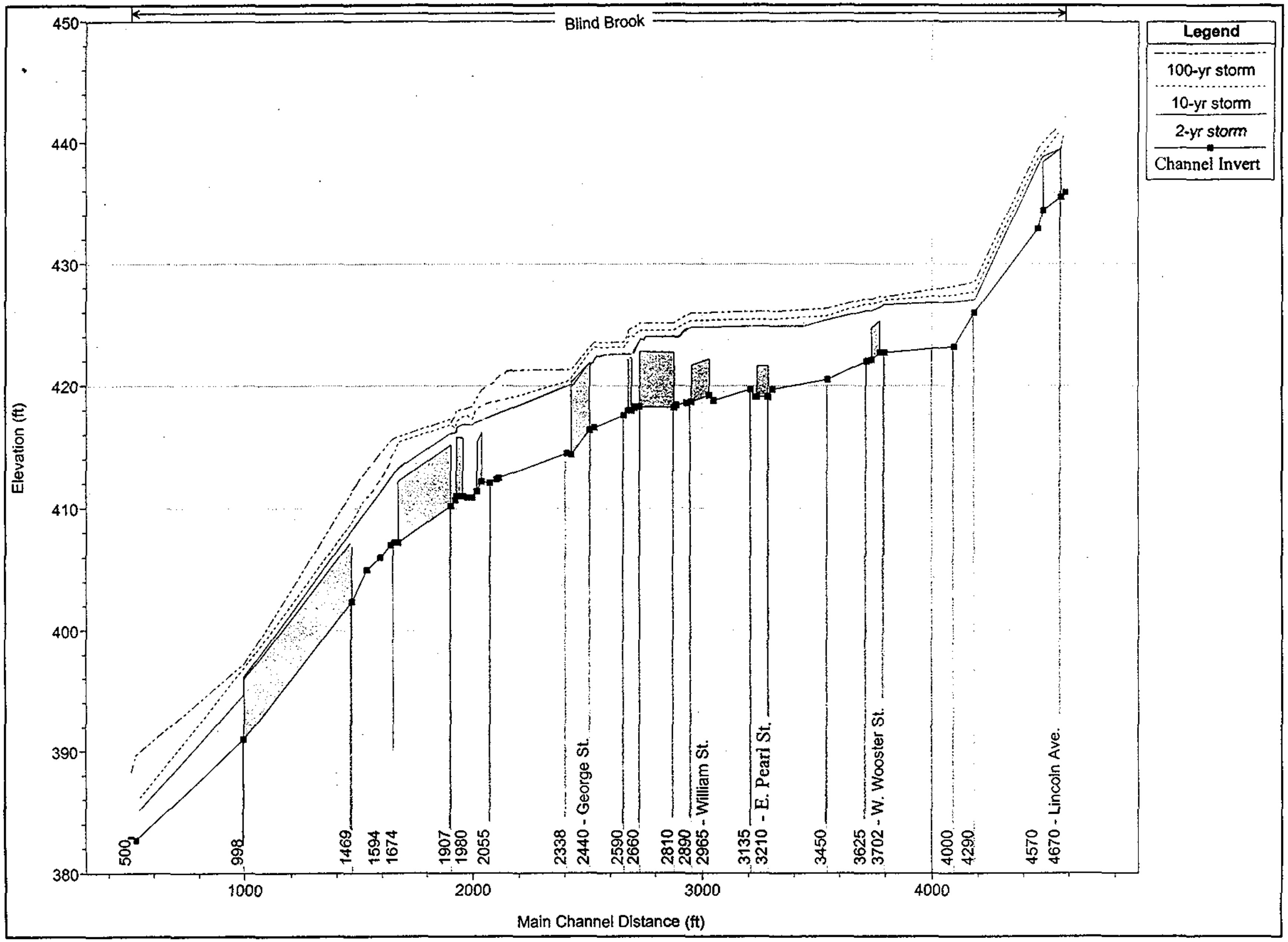


PLATE 10
20YR, 10-YR, AND 100-YR
WATER SURFACE PROFILE
BLIND BROOK - DANBURY, CT

APPENDIX A

HEC-HMS SUMMARY OF RESULTS

HMS * Summary of Results

Project : danbury   Run Name : Run 32

Start of Simulation	:	0000	Basin Model	: Existing Cond2
End of Simulation	:	2400	Precip Model	: 2 yr/12 hour
Execution Time	:	1734	Control Specs	: Control 1

Hydrologic Element	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)	Drainage Area (sq mi)
4	56.289	0635	28.131	0.21
Upstream Wooster	26.789	0715	28.234	0.21
1- Parks Pond	6.6885	1200	9.6100	0.2
Parks Pond Dam	6.5396	1215	9.6841	0.2
2 - Tarrywile Lake	17.305	0705	10.818	0.50
Tarrywile Pond Dam	10.479	1230	10.781	0.50
Junction 1	16.983	1220	20.465	0.70
3	104.79	0635	39.578	0.35
Upstream Jefferson	84.320	0655	60.073	1.05
Junc. 2 Wooster St.	110.43	0655	88.307	1.26
6	33.591	0630	22.489	0.034
5	76.739	0630	30.943	0.095
Junc. 3 West St.	193.50	0630	141.74	1.389
7	27.156	0635	21.106	0.040
Junc. 4 - d/s limit	220.61	0630	162.84	1.429

HMS * Summary of Results

Project : danbury   Run Name : Run 34

Start of Simulation :   0000   Basin Model   : Existing Cond2
End of Simulation   :   2400   Precip Model   : 5yr/12 hour
Execution Time   :   1735   Control Specs : Control 1

Hydrologic Element	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)	Drainage Area (sq mi)
4	88.797	0640	31.853	0.21
Upstream Wooster	34.837	0725	31.953	0.21
1- Parks Pond	11.839	0700	10.896	0.2
Parks Pond Dam	9.2481	0815	10.970	0.2
2 - Tarrywile Lake	47.890	0705	17.997	0.50
Tarrywile Pond Dam	17.381	0940	17.123	0.50
Junction 1	26.179	0910	28.093	0.70
3	165.06	0635	51.774	0.35
Upstream Jefferson	148.30	0650	79.718	1.05
Junc. 2 Wooster St.	181.22	0650	111.67	1.26
6	43.667	0630	23.577	0.034
5	104.82	0630	35.393	0.095
Junc. 3 West St.	290.55	0635	170.64	1.389
7	36.795	0635	22.132	0.040
Junc. 4 - d/s limit	327.35	0635	192.77	1.429

HMS * Summary of Results

Project : danbury   Run Name : Run 28

Start of Simulation	:	0000	Basin Model	: Existing Cond2
End of Simulation	:	2400	Precip Model	: 10yr/12hour
Execution Time	:	1529	Control Specs	: Control 1

Hydrologic Element	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)	Drainage Area (sq mi)
4	121.16	0635	39.081	0.21
Upstream Wooster	47.649	0720	39.031	0.21
1- Parks Pond	24.738	0655	12.873	0.2
Parks Pond Dam	15.606	0740	12.947	0.2
2 - Tarrywile Lake	87.100	0705	28.229	0.50
Tarrywile Pond Dam	27.546	0915	26.079	0.50
Junction 1	41.191	0830	39.026	0.70
3	223.35	0635	65.075	0.35
Upstream Jefferson	221.10	0645	103.75	1.05
Junc. 2 Wooster St.	257.83	0645	142.78	1.26
6	51.615	0630	24.754	0.034
5	126.99	0630	40.704	0.095
Junc. 3 West St.	388.93	0640	208.23	1.389
7	45.258	0635	23.294	0.040
Junc. 4 - d/s limit	431.94	0640	231.53	1.429

HMS * Summary of Results

Project : danbury   Run Name : Run 33

Start of Simulation :   Basin Model : Existing Cond2
End of Simulation :   Precip Model : 25yr/12hour
Execution Time :   Control Specs : Control 1

Hydrologic Element	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)	Drainage Area (sq mi)
4	153.30	0635	46.793	0.21
Upstream Wooster	86.007	0705	46.510	0.21
1- Parks Pond	41.874	0650	15.375	0.2
Parks Pond Dam	24.824	0730	15.449	0.2
2 - Tarrywile Lake	133.57	0700	40.405	0.50
Tarrywile Pond Dam	42.579	0850	36.772	0.50
Junction 1	63.487	0815	52.221	0.70
3	279.74	0635	78.761	0.35
Upstream Jefferson	290.94	0645	130.16	1.05
Junc. 2 Wooster St.	354.16	0650	176.67	1.26
6	58.356	0630	25.937	0.034
5	145.82	0630	45.738	0.095
Junc. 3 West St.	505.42	0635	248.34	1.389
7	52.944	0635	24.500	0.040
Junc. 4 - d/s limit	558.36	0635	272.84	1.429

HMS * Summary of Results

Project : danbury   Run Name : Run 29

Start of Simulation	:	0000	Basin Model	: Existing Cond2
End of Simulation	:	2400	Precip Model	: 50yr/12 hour
Execution Time	:	1531	Control Specs	: Control 1

Hydrologic Element	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)	Drainage Area (sq mi)
4	191.01	0635	58.375	0.21
Upstream Wooster	127.99	0655	57.274	0.21
1- Parks Pond	67.242	0650	22.161	0.2
Parks Pond Dam	48.374	0710	22.076	0.2
2 - Tarrywile Lake	198.98	0700	60.811	0.50
Tarrywile Pond Dam	65.407	0845	55.040	0.50
Junction 1	97.082	0740	77.116	0.70
3	343.52	0635	99.026	0.35
Upstream Jefferson	368.18	0645	173.90	1.05
Junc. 2 Wooster St.	482.42	0645	231.17	1.26
6	64.166	0630	27.808	0.034
5	162.07	0630	52.439	0.095
Junc. 3 West St.	650.81	0640	311.42	1.389
7	60.743	0630	26.467	0.040
Junc. 4 - d/s limit	709.05	0635	337.89	1.429

HMS * Summary of Results

Project : danbury Run Name : Run 19

Start of Simulation	:	0000	Basin Model	:	Existing Cond2
End of Simulation	:	2400	Precip Model	:	100yr/12 hour
Execution Time	:	1531	Control Specs	:	Control 1

Hydrologic Element	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)	Drainage Area (sq mi)
4	212.27	0635	61.653	0.21
Upstream Wooster	144.47	0655	60.118	0.21
1- Parks Pond	78.087	0650	23.942	0.2
Parks Pond Dam	57.663	0710	23.782	0.2
2 - Tarrywile Lake	227.23	0700	65.864	0.50
Tarrywile Pond Dam	73.496	0835	59.287	0.50
Junction 1	111.34	0735	83.069	0.70
3	381.41	0635	104.79	0.35
Upstream Jefferson	405.57	0645	185.01	1.05
Junc. 2 Wooster St.	542.12	0650	245.13	1.26
6	69.447	0630	28.094	0.034
5	176.79	0630	54.578	0.095
Junc. 3 West St.	720.20	0635	327.80	1.389
7	66.135	0630	26.772	0.040
Junc. 4 - d/s limit	786.21	0635	354.57	1.429

HMS * Summary of Results

Project : danbury   Run Name : Run 23

Start of Simulation :	0000	Basin Model	: Existing Cond2
End of Simulation :	2400	Precip Model	: 100yr/24hour
Execution Time :	1531	Control Specs	: Control 1

Hydrologic Element	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)	Drainage Area (sq mi)
4	207.35	1235	60.183	0.21
Upstream Wooster	149.75	1255	58.195	0.21
1- Parks Pond	87.650	1245	26.105	0.2
Parks Pond Dam	69.909	1310	25.819	0.2
2 - Tarrywile Lake	251.04	1300	65.600	0.50
Tarrywile Pond Dam	89.116	1435	56.538	0.50
Junction 1	134.73	1335	82.357	0.70
3	368.71	1235	98.575	0.35
Upstream Jefferson	415.24	1245	177.68	1.05
Junc. 2 Wooster St.	562.44	1250	235.87	1.26
6	63.471	1230	29.745	0.034
5	161.12	1230	52.579	0.095
Junc. 3 West St.	728.50	1240	318.19	1.389
7	62.107	1230	28.543	0.040
Junc. 4 - d/s limit	787.69	1240	346.74	1.429

HMS * Summary of Results

Project : danbury Run Name : Run 21

Start of Simulation	:	0000	Basin Model	: Existing Cond2
End of Simulation	:	2400	Precip Model	: Floyd/24hr/9.66in
Execution Time	:	1531	Control Specs	: Control 1

Hydrologic Element	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)	Drainage Area (sq mi)
4	161.29	1905	80.527	0.21
Upstream Wooster	148.20	1925	76.221	0.21
1- Parks Pond	101.84	1915	41.753	0.2
Parks Pond Dam	96.374	1935	41.192	0.2
2 - Tarrywile Lake	275.85	1925	107.80	0.50
Tarrywile Pond Dam	204.66	2130	81.128	0.50
Junction 1	290.44	2115	122.32	0.70
3	275.24	1905	131.32	0.35
Upstream Jefferson	490.53	2105	248.36	1.05
Junc. 2 Wooster St.	617.39	2105	324.59	1.26
6	41.115	1900	33.942	0.034
5	99.789	1900	63.254	0.095
Junc. 3 West St.	730.84	1910	421.78	1.389
7	43.632	1900	33.197	0.040
Junc. 4 - d/s limit	773.48	1910	454.98	1.429